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27.1 GENERAL

Bridges supported in the conventional way by abutments and piers require bearings to transfer girder reactions without overstressing the supports insuring that the bridge functions as intended. In general, bridges require bearings that are more elaborate than those required for building columns, girders and trusses. Bridge bearings require more consideration in minimizing forces caused by temperature change, friction, and restraint against elastic deformations. A more detailed analysis in bridge bearing design considers the following:

1. Bridges are usually supported by reinforced concrete substructure units and the magnitude of the horizontal thrust determines the size of the substructure units. The coefficient of friction on bridge bearings should be as low as practicable.

2. Bridge bearings must be capable of withstanding and transferring the dynamic forces and the resulting vibrations without causing eventual wear and destruction of the substructure units.

3. Most bridges are exposed to the elements of nature. Bridge bearings are subjected to more frequent and greater total expansion and contraction movement due to changes in temperature than those required by buildings. Since bridge bearings are exposed to the weather, they are designed as maintenance free as practical.

4. It is often necessary to jack girders to rehabilitate bearings. Where possible, provide an area 6” by 7” (150 x 175 mm) that is at least 4” (100 mm) below the girder to perform future maintenance. Do not create unnecessary expenses as contractors are innovative when required to provide maintenance platforms.

27.2 BEARING TYPES

Bridge bearings are of two general types, expansion and fixed. The expansion bearings provide for rotational movements of the girders as well as longitudinal movement for the expansion and contraction of the bridge spans. If an expansion bearing develops a large resistance to longitudinal movement due to corrosion or other causes, this frictional force opposes the natural expansion or contraction of the span creating a force within the span that could lead to a maintenance problem in the future. The fixed bearing acts as a hinge by permitting rotational movement while at the same time preventing longitudinal movement. The function of the fixed bearing is to prevent the superstructure from moving longitudinally off of the substructure units. Both expansion and fixed bearings transfer lateral forces such as wind and centrifugal loading from the superstructure to the substructure units. Both bearing types are set parallel to the direction of structural movement; bearings are not set parallel to flared girders.

(1) Steel Bearings

For short to intermediate span lengths rocker plates are used for both expansion and fixed bearings to permit girder rotation. The required length of the rocker plate is determined from the allowable line bearing stress given in Section 10 of the AASHTO specifications.

The rocker plate is set on a masonry plate which transfers the girder reaction from the rocker to the substructure unit. The masonry plate is attached to the substructure unit with anchor bolts. Pintles set into the masonry plate prevent the rocker from sliding off the masonry plate while allowing rotation to occur.

For the expansion bearings, two additional plates are employed, a top plate and a teflon plate. Current experience indicates that a stainless steel top plate reduces corrosion activity and is the recommended alternate to steel. The top plate is set on top of a teflon plate allowing expansion and contraction to occur. A Type "A-T" bearing selection example is given in Section 27.6 of this Chapter. For details refer to Standard 27.2 for steel girders and Standard 27.9 for pretensioned girders.

For long span bridges having girder reactions of 400 kips (1780 kN) or greater a built-up bearing is recommended to allow for greater longitudinal movements and reduced coefficients of friction. Current practice for Type "B" expansion bearings is to employ a 4 percent maximum and a 2 percent minimum coefficient of friction value for design. A Type "B" bearing design example is given in Section 27.5. Refer to Standards 27.3 and 27.4 for details. The expansion bearing has a curved rocker plate and a curved upper web plate edge coinciding with the center of curvature of the rocker plate permitting rotation of the entire bearing assembly thus reducing friction forces. Also, the fixed bearing has a curved upper web plate edge to allow for girder rotation. Web stiffener plates are used to prevent local web buckling for both fixed and expansion bearings. Bearing details for fixed abutments are given on Standard 27.5.

(2) Elastomeric Bearings

Elastomeric bearings are either fabricated as plain bearing pads (consisting of elastomer only) or as laminated (steel reinforced) bearings (consisting of alternate layers of steel reinforcement and elastomer bonded together). These bearings are designed to transmit loads and accommodate movements between a bridge and its supporting structure. Performance information indicates that elastomeric bearings are functional and reliable when designed within the structural limits of the material. See AASHTO Section 14 (Division I) and Section 18 (Division II) for design and construction requirements of elastomeric bearings.

For several years plain elastomeric bearing pads have performed well on prestressed girder structures. Refer to Standard 19.14 for prestressed girder bearing pad details. Prestressed girders using this detail are fixed into the concrete diaphragms at the supports and the girders are set on 1/2" (13 mm) thick plain elastomeric bearing pads.

Laminated (steel reinforced) bearings can be designed by "Method A" as outlined in AASHTO 14.6.6 and NCHRP-248 or by "Method B" as shown in AASHTO 14.6.5 and NCHRP-298. The Bridge Office currently uses "Method A". The design is based on service loads without impact.

The definition for shear deformation (Δs) of the bearing states that its contribution from thermal effects are computed between the installation temperature and the least favorable extreme temperature. In NCHRP 20-07/106, the installation temperature used for designing elastomeric bearings supporting concrete bridges is defined. As a result of this report, Bridge Standard 27.7 (for prestressed girders) is based on a design installation temperature of 60°F (15°C). The maximum design temperature range for prestressed girder structures is (60°F-5°F) = 55°F (30°C). The shear deformation due to thermal effects (Δs_T) equals (Expansion length) (55°F) (0.0000060 ft/ft/°F) for prestressed girder structures. Shear deformation due to creep/shrinkage effects ($\Delta s_{cr/sh}$) should be added to (Δs_T) for prestressed girder structures. The value for ($\Delta s_{cr/sh}$) equals (Expansion length) (0.0003 ft/ft). The combined effect of using a design temperature range of (55°F) and a creep/shrinkage coefficient of (0.0003 ft/ft) is equivalent to using only a design temperature range of (105°F). This approximates the previous approach that used a design temperature range of (100°F). AASHTO requires the total thickness of all elastomeric layers in the bearing to be twice the total shear deformation to avoid rollover at the edges and prevent delamination.

A preliminary value for bearing height (H), based on expansion length, can be found on Standard 27.7 for prestressed girder structures. The corresponding bearing length (L) based on stability requirements can be found there also. The bearing width (W) is then chosen as the bottom flange width minus 2" (50 mm) and is checked against stability requirements. Using values for H, L and W just selected, the AASHTO requirements for compressive stress, compressive deflection, steel reinforcement thickness, rotation and anchorage can be checked and the preliminary values adjusted as required.

Note: AASHTO does not permit tapered elastomer layers in reinforced bearings.

Laminated (steel reinforced) bearings must be placed on a level surface otherwise gravity loads will produce shear strain in the bearing due to inclined forces. The angle between the alignment of the underside of the girder (due to the slope of the gradeline, camber and dead load rotation) and a horizontal line must not exceed (0.01 radians), per AASHTO14.7.2. If the angle is greater than (0.01 radians), the 1 1/2" (38 mm) top steel plate must be tapered to provide a level load surface along the bottom of this plate under these conditions. The tapered plate will have a minimum thickness of 1 1/2" (38 mm). The angle between the alignment of the underside of the girder (due to the slope of the gradeline, the rotation of the girder due to dead load plus live load, and camber) and the alignment of the bottom of the bearing must not exceed the allowable rotation angle (θ_m), as per AASHTO 14.6.6.3.5, when a tapered plate is not used. If a tapered plate is used, the angle between the alignment of the underside of tapered plate (due to live load rotation) and the alignment of the bottom of the bearing (due to construction tolerances) must not exceed the allowable rotation (θ_m).

Reinforced laminated bearing details, steel plate and elastomer thicknesses are given on Standard 27.7 for prestressed concrete girders. Refer to Section 27.7 of this chapter for a laminated (steel reinforced) elastomeric bearing design example.

(3) TFE Bearing Surface

These bearings are designed to translate or rotate by sliding a self-lubricating polytetrafluoroethylene (TFE) surface across a smooth, hard mating surface preferably of stainless steel or other equally corrosive resistant materials. Expansion bearings of teflon are not used without provision for rotation. A rocker plate or layer of elastomer is provided to facilitate rotation due to live load deflection or change of camber. The teflon sliding surface must be bonded to a rigid back-up material capable of resisting horizontal shear and bending stresses to which the sliding surfaces may be subjected.

Design and construction requirements for TFE bearing surfaces are given in Sections 14 and 18 of the AASHTO specifications for highway bridges, respectively. Stainless steel-TFE expansion bearing details are given on Standard 27.8. The bearing is referred to as Type "A-T".

TFE can be made into different shapes and forms, its use as a bearing material is suited to many different types of expansion bearings. Many combinations of teflon bearings and backing materials are commercially available. Generally unfilled TFE is specified for the Type "A-T" bearings. Friction values are given in the AASHTO Specifications; they vary with loading and the detail used for TFE.

(4) Pot Bearings

The pot bearing was developed in 1959 as an alternate to heavy steel bearings. The bearing consists of a circular non-reinforced neoprene or rubber pad, of relatively thin section, which is totally enclosed by a steel pot. The rubber is prevented from bulging by the pot containing it and acts similar to a fluid under high pressure. The result is a bearing providing suitable rotation and at the same time giving the effect of a point-contact rocker bearing since the center of pressure does not vary more than 4 percent.

Although experience has shown the pot bearing to be compact and efficient; it currently is not cost competitive with the type "B" steel bearing on a multi-girder structure. A final remaining concern is the fact that satisfactory rotational operation of the bearing is not achieved until at least 25 percent of the working load is applied; therefore, additional consideration must be given to specifying the erection procedures.

27.3 STEEL BEARING DESIGN CONSIDERATIONS

Design considerations are presented for bearing types "A-T" and "B" bearings as shown in Standards 27.2 through 27.4 and 27.8. The type "A-T" bearings are designed to comply with the latest AASHTO Specification requirements for anchor bolts. The type "B" bearings are designed with A709 Grade 50 steel and a bearing reaction capacity range of 400-1500 kips (1780 to 6670 kN). The principal application of type "B" bearings is for long multigirder spans or two-girder systems having reactions of 400 kips (1780 kN) or greater and a requirement of smaller longitudinal forces on the substructure units. Since strength is not a governing criteria, both type "A-T" and "B" bearing anchor bolts are design with Grade 36 (250) steel.

(1) Type "A-T" Bearings

The design of type "A-T" bearings is relatively simple. The first consideration is the rocker plate length which is proportional to the allowable line bearing based on a radius of 24" (610 mm) using Grade 50W (345W) steel. The rocker plate thickness is determined from a minimum of 1 1/2" (38 mm) to a maximum computed from the moment by assuming one-half the bearing reaction value (N/2) acting at a lever arm of one-fourth the width of the teflon coated plate (W/4) over the length of the rocker plate. The teflon coated plate is designed with a minimum width of 7" and the allowable stress of 1500 psi for dead load and 2500 psi for all loads on the gross area; in many cases this controls the capacity of the expansion bearings as given in Standard 27.8.

The design of the masonry plate is based on a maximum allowable bearing stress of 1 ksi (6.9 MPa) on the concrete masonry. The masonry plate thickness is determined from the maximum bending moments about the x-or y-axis using a uniform pressure distribution of 1 ksi (6.9 MPa).

(2) Type "B" Bearings

The first design revision for the type "B" bearing is to increase the "A" dimension to allow for a higher bending capacity. Previous design allowed the sole plate to transfer the reaction in direct loading. However, current practice is to use two pairs of bearing stiffeners, 9" (225 mm) apart, at the piers. As a result, the sole plate is more severely loaded in bending. Due to the thickness of the sole plate providing the possibility for stress redistribution and the partial interaction with the lower girder flange in bending, an over-stress of 50 percent is used in the design of the sole plate. In order to accommodate the additional bending stress, the "A" dimension is increased and the "B" dimension is decreased as shown in Standard 27.3 and 27.4. Contact between the sole plate and web is considered as full bearing and not as line bearing.

Bearing web and stiffeners are designed for combined axial and bending stresses resulting from maximum bearing load at maximum movement. The counter-bending moment from friction is conservatively neglected. The fillet weld size at the web-rocker plate intercept is also designed considering the combined stresses resulting from maximum bearing load and movement. For fixed shoe bearings, the fillet weld size at the web-masonry plate intercept is designed for maximum stresses due to both lateral and longitudinal forces. An overstress of 25 percent is used for the design of the bearing web and stiffeners and the fillet weld size at the web-rocker or web-masonry plate intercept in accordance to AASHTO specification, under Group V loading.

The rocker plate design is very similar to the type "A" bearing. The length is proportional to the allowable line bearing stress between the rocker and masonry plate. The allowable line bearing is directly proportional to the rocker plate radius and the yield strength of the steel. Radii are selected on the basis of bearing capacity, thermal movement, and bearing geometry limitations. In order to prevent vertical movement as the bearing rotates, the rocker and top of web plate have the same center of curvature. The rocker plate thickness is determined from the bending moment computed as described for type "A" bearings.

The rocker bearing is set vertical at 45°F (7°C). This is the mean temperature for the range of thermal movement for both concrete and steel structures.

The masonry plate design for the type "B" expansion rocker bearing is based on an allowable concrete masonry bearing stress of 800 psi (5.5 MPa) as shown in Section 27.5. This value is selected out of an allowable range of 700-1000 psi (4.8-6.9 MPa) in lieu of more detailed analysis for eccentric loading and possible masonry plate edge yielding. In order to compute the masonry plate length, L , an assumed width, K, must be chosen. The masonry plate design for the type "B" fixed shoe bearing is based on an allowable concrete masonry bearing stress of 1 ksi (6.9 MPa). This is in accordance with AASHTO specifications for shoes employing a hinge. For type "B" fixed shoe bearing masonry plate thickness computations, refer to the filed documentation for analysis and design assumptions.

An alternate bearing design was considered during the revision of type "B" bearing standards. A rocker bearing incorporating a solid pin was sent out to fabricators for comments and cost comparison with the standard type "B" rocker bearing. The cost of the alternate bearing was 30 to 35 percent higher than our present standard bearing. Design and documentation are on file in the Development Unit for future reference.

Based on steel fabricator's recommendations a minimum number of plate thicknesses over 2" (50 mm) were employed. This will alleviate stocking or special ordering several sizes of plates.

Plate thicknesses required in the Standards are specified 1/16" (2 mm) less than the rolled thickness to allow for fabrication milling of the plates to flat surfaces. Most plates attain a small amount of warpage from the rolling process.

27.4 HOLD DOWN DEVICES

Details for hold down devices are given in Standard 27.6. The design criteria for employing hold down devices is located in Bridge Manual Chapter 24 - Steel Girder Structures.

27.5 DESIGN EXAMPLE - TYPE "B" ROCKER BEARING

| English units are used for this example.

| A design example for the rocker bearing is given to show how the dimensions in Standard 27.3 were arrived at and to assist the designer if a bearing capacity greater than 1500 kips is required.

(1) Sole Plate Design

Given the following values: N = 1200 kips, G = 2'-3, K = 2'-6, and R = 1'-9.

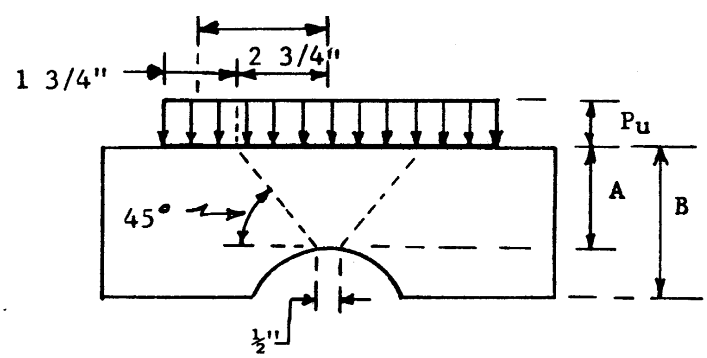
A uniform load distribution over a width equal to 9" is used due to the paired stiffener spacing; the sole plate uniform loading is equal to:

$$Pu = \frac{N}{(G)(9")} \quad \text{where}$$

N equals the bearing reaction and G equals the length of the sole plate.

$$Pu = \frac{1200^k}{(27")(9")} = 4.94 \text{ k/in}^2$$

An assumed web to sole plate contact width of 1/2 inch is used and the approximation that the load spreads through the sole plate at an angle of 45 degrees as shown below. The uniform loading within the dashed lines is transferred from the sole plate to the web in direct axial stress.



Sole Plate Loading

The value of "A" is assumed as 2 1/2 inches and for geometric reasons the "B" dimension is 4 inches. The "A" dimension is used to determine the width of the load transferred in direct stress and checked in bending from the moment of the loading transferred in bending about the centerline of the sole plate. Using a 1 inch strip, the moment, M, is as follows:

M = (1.75")(Pu)(X) where X equals

(5.5 + 1.75)/2 = 3.625"

M = (1.75")(4.94 k/in)(3.625") = 31.34 in-k.

The required sole plate thickness, "A", using an overstress of 50 percent is:

$$A = (6M/1.5\ F_s)^{1/2}$$

$$A = (6(31.34\ \text{in.-k})/(1.5)(27.0\ \text{ksi}))^{1/2}$$

A = 2.16". <u>USE: A = 2 1/2"</u>

(A) Contact Stresses

Contact stresses are included for informational purposes and are not part of the design procedure.

The contact width, b, and the maximum contact stress, S_c, are computed from "Formulas for Stress and Strain" by R. J. Roarke, pages 318 through 329. Assuming the modulus of elasticity, $E_1 = E_2$, and Poisson's ratio $V_1 = V_2 = 0.30$, then the contact width equals:

$$b = 2.15(P/E(D_1D_2/D_1\text{-}D_2))^{1/2} \quad \text{where}$$

P equals the load per linear inch, D_1 equals the diameter of the sole plate, and D_2 equals the diameter of the web plate.

$$P = \frac{N}{G} = \frac{1200^k}{27"} = 44.44\ \text{k/in}$$

If E = 29,000 ksi,

D_1 = 4.406" & D_2 = 4.375";

$$b = 2.15(44.44(4.406)(4.375)/29000(4.406\text{-}4.375))^{1/2}$$

$b = 2.10" > 1/2"$ assumed for sole plate comp.

$S_c(max) = 0.591(PE(D_1\text{-}D_2)/D_1D_2)^{1/2}$ from Roarke:

$S_c(max) = .591(44.44(29000)(4.406\text{-}4.375)/(4.406)(4.375))^{1/2}$

$S_c(max) = 26.91$ ksi.

The actual contact stress equals:

$$S_c(act) = \frac{N}{(G)(b)}$$

$$S_c(act) = \frac{1200^k}{(27")(2.10")} = 21.16 \text{ ksi* which}$$

for comparison is less than $S_c(max)$.

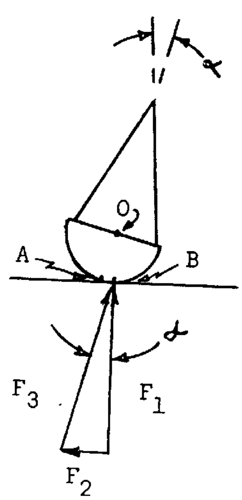
(2) Web and Stiffener Design

Given the following information: Maximum rocker movement,

AB = 5", C = 4", D = 1'-6, and E = 4".

Refer to the following sketch for bearing movement and resultant forces.

* AASHTO Specifications allow up to $0.40F_y$ bearing stress on rockers and pins subject to rotation.



$$\alpha = \frac{(AB)(180)}{(R)(\pi)}$$

$$\alpha = \frac{5"}{21"}(57.3°) = 13.64°$$

$F_1 = 1200^k$

$F_2 = (1200)(\sin 13.64°)$

$F_2 = 283.0^k$

$F_3 = (1200)(\cos 13.64°)$

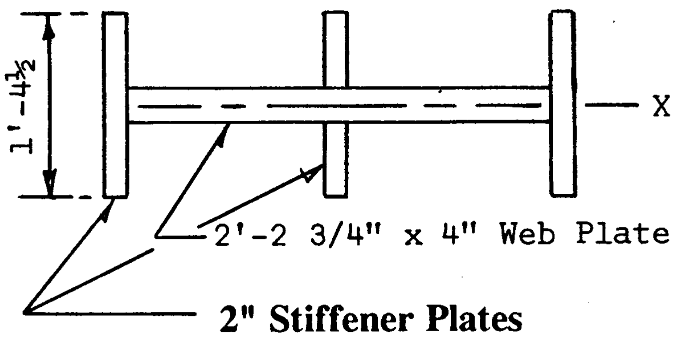
$F_3 = 1166.2^k$

The summation of moments about point 0, neglecting the variable friction component, equals:

$$M_o = (5.0")(1166.2^k) - (4.0")(283.0^k)$$

$$M_o = 5831.0 - 1132.0 = 4699.0 \text{ in-k.}$$

The available section modulus at the web-rocker plate intercept, assuming 2 inch thick stiffeners, is as follows:



$$I_x = (26.75)(4)^3/(12) + (2)(2)(16.5)^3/(12) + (2)(16.5^3-4^3)/12$$

$$I_x = 142.67 + 1497.38 + 738.02 = 2378.07 \text{ in}^4$$

$$S_x = \frac{2378.07}{8.25} = 288.2 \text{ in}^3$$

$$\text{Area} = (26.75)(4) + (2)(2)(16.5) + (2)(2)(6.25) = 198.0 \text{ in}^2$$

The combined axial and bending stress, f_s, equals:

$$f_s = \frac{F_3}{A} \pm \frac{M_0}{S_x}$$

$$f_s = \frac{1166.2}{198.0} \pm \frac{4699.0}{288.2} = 5.9 \text{ ksi} \pm 16.3 \text{ ksi}$$

$f_s(max)$ = 22.2 ksi (C) Less than $1.25F_s$

$f_s(min)$ = 10.4 ksi (T) Less than $1.25F_s$

<u>USE: 4" Web and 2" Stiffener Thickness</u>

(3) Weld Design at Web-Rocker Intercept

The fillet weld size will be determined using E-70 electrodes and an overstress of 25 percent. The overstress is allowing for weld metal at the web-rocker intercept using the same reasoning as discussed in Section 27.3(2) of this chapter for the steel overstress. The allowable stress for the weld is 14,700 psi as recommended by AASHTO, Section 7, Article 1.7.2.

The approximate fillet weld length, L_w, equals:

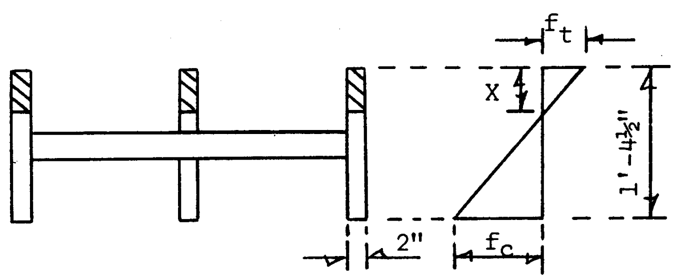
$L_w = (2)(26.75) + (2)(16.5) + (2)(12.5) + (6)(2) + (4)(6.25)$

$L_w = 148.5$ in

The shear component of stress is:

$$f_v = \frac{F_2}{L_w} = \frac{283.0^k}{148.5''} = 1.91 \text{ k/in.}$$

The bending moment component for stress is computed from the combined stresses; refer to the following diagram and computations.



By similar stress triangles the following equation is:

$$\frac{f_t}{X} = \frac{f_c}{16.5 - X}$$

Where f_t = 10.4 ksi and f_c = 22.2 ksi; solving for X,

10.4(16.5 - X) = X(22.2),

171.6 - 10.4X = 22.2X,

X = 5.26".

The weld metal must resist the following force.

$F = (3)(2")(10.4\text{ ksi})(5.26")/(2) = 164.1^{k}$

and the resulting moment of

$M = (164.1^{k})(5.26")(2/3) = 575.4$ in-k.

The moment of inertia of the weld lines is computed as follows:

$I = (6)(5.26")^3/(3) + (3)(2")(5.26")^2$

$I = 291.1 + 166.0 = 457.1\text{ in}^3$

The bending moment component, f_b, equals:

$$f_b = \frac{(575.4\text{ in-k})(5.26")}{457.1\text{ in}^3} = 6.62\text{ k/in.}$$

The resultant stress, f_r, is equal to:

$f_r = (f_v)^2 + (f_b)^2)^{1/2} = (1.91)^2 + (6.62)^2)^{1/2} = 6.89$ k/in.

The weld size, W_s, is obtained from the allowable weld metal stress as follows:

$(0.707)(14.7\text{ ksi})(1.25)(W_s) = 6.89$ k/in

W_s = 0.53 in. Refer to Std. 27.3

<u>USE: 3/4" Fillet Weld Size</u>

(4) Rocker Plate Design

The required rocker plate length, F, is determined from the equation:

$$F = \frac{N}{P} + 2\text{ (Pintle dia.) where}$$

P is the allowable line bearing and N is the bearing reaction. The allowable line bearing is dependent on the allowable yield stress, F_y, and the diameter, d, of the rocker plate. Refer to AASHTO, Section 7, Article 1.7.4 for the following equations:

$$\text{If } d \leq 25'',\ P = \frac{F_y - 13000}{20000}(600\,d) \text{ and}$$

$$\text{If } 25'' > d \leq 125'',\ P = \frac{F_y - 13000}{20000}(3000\ \sqrt{d}).$$

The allowable line bearing value, P, is computed assuming d = 42 inches, and for A588 steel, F_y = 50 psi.

$$P = \frac{50000 - 13000}{20000}(3000\,(42))^{1/2} = 35.97\text{ k/in}$$

Referring to the previous equation for the required rocker plate length, F, equals:

$$F = \frac{1200^k}{35.97\text{ k-in}} + 2(2\ 1/2'')$$

F = 33.4 + 5.0 = 38.4"; however,

F ≥ G + 6" = 2'-3 + 6" = 2'-9.

USE: 3'-3 Line Bearing

The rocker plate thickness is computed by assuming the reaction, N/2, acts at an eccentricity of one-fourth the rocker width, D/4, from its centerline. The bending moment is equal to (N/2)(D/4) and the section modulus is equal to $(F)(E)^2/(6)$ where E is the required rocker plate thickness.

If $S = \frac{M}{F_s}$ by substituting

$$\frac{(F)(E)^2}{6} = \frac{(N/2)(D/4)}{F_s} \text{ then,}$$

$$(E)^2 = \frac{6(N)(D)}{8(F)(F_s)},$$

$$E = (3(1200)(18)/(4)(39)(27))^{1/2} = 3.92"$$

USE: 4" rocker Plate Thickness

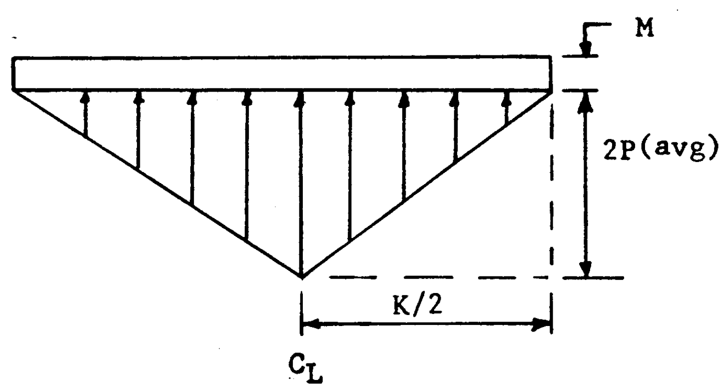
(5) Masonry Plate Design

The masonry plate dimensions are computed neglecting the small effect of the area loss due to pintle and anchor bolt holes. The masonry plate length, L, is designed on the basis of an assumed width, K, and a uniform pressure distribution of load between the plate and concrete. Using an allowable concrete masonry bearing stress of 800 psi, the required plate length is:

$$L = \frac{N}{(0.8)(K)} = \frac{1200^k}{(0.8)(30")} = 50.0".$$

USE: 4'-2 Masonry Plate Length

The masonry plate thickness, M, in the longitudinal direction of the structure, is based on a pressure diagram equal to 2P average triangular at the centerline of the masonry plate. The triangular pressure diagram is used as a better approximation of the load distribution under the rocker plate due to possible yielding of the masonry plate edges. The pressure diagram is given as follows.



The maximum effective length, L_r, for masonry plate thickness design is equal to:

$$L_r = F + 3M$$

where M is approximated for the first trial computation, L_r must also be equal to or less than L, the actual masonry plate length.

A trial value of M = 4", gives

$$L_r = 3'\text{-}3 + 3(4") = 4'\text{-}3 > L \quad \text{N.G.}$$

Therefore, $L_r = L = 4'\text{-}2$.

The value of $P(avg) = \dfrac{N}{KL_r}$ and from the pressure diagram

The bending moment, M_b, equals:

$$M_b = (1/2)(2P(avg))(K/2)(K/6)(L_r)$$

$$M_b = (1/12)(P(avg))(K^2)(L_r)$$

For a rectangular shape, the section modulus, S, equals:

$$S = (M^2)(L_r)/(6).$$

Substituting into the flexural equation.

$$S = \frac{M_b}{F_s},$$

$$\frac{(M^2)(L_r)}{6} = \frac{(1/12)(P(avg))(K^2)(L_r)}{F_s},$$

substituting N/KL_r for P(avg) and dividing out common terms, the plate thickness is:

$$M = (NK/2F_sL_r)^{1/2}$$

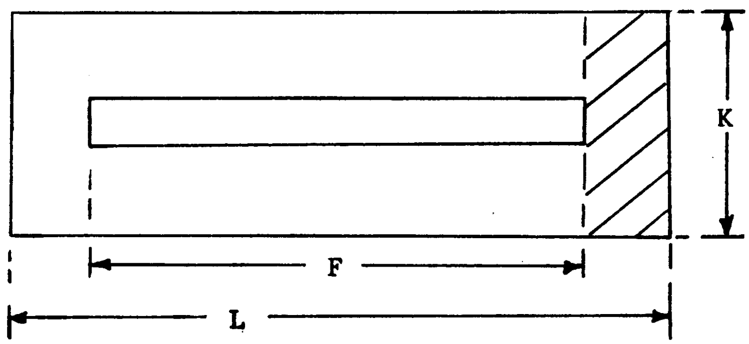
$$M = (1200^k(30")/2(27\text{ ksi})(50"))^{1/2} \qquad = 3.65"$$

<u>USE: 4" Masonry Plate Thickness</u>

The end projection beyond the rocker plate is also checked for bending using a uniform pressure distribution equal to:

$$P(avg) = \frac{N}{KL} = \frac{1200^k}{(30")(50")} = 0.80 \text{ ksi} .$$

Refer to the following diagram for end projection.



End Projection = (L-F)/2 = (50-39)/2 = 5.5" = L_e

The bending moment, $M_b = (1/2)(P(avg))(K)(L_e)^2$ and the section modulus equals:

$$S = \frac{KM^2}{6}$$

Solving for M in the flexural equation gives:

$$M = (3.0(P(avg))(L_e^2)/F_s)^{1/2}$$

$$M = (3.0(0.80 \text{ ksi})(5.5")^2/27 \text{ ksi})^{1/2}$$

The previous masonry plate thickness requirement of 4 inches governs.

(6) Pintle Design

Assume a reaction of 1,400 kips, an average depth of 18 feet, and a span length of 360 feet.

From AASHTO Specifications, transverse wind loading is given as 50 pounds per square foot for girder type structures. The total transverse wind force is equal to (.050) (18)(360) = 324 kips. Assume one half of the total force is transferred to the remaining girders and that a full bearing type pintle connection is used with an allowable shear of 20 ksi.

The required pintle area = 162 kips/20 ksi = 8.1 square inches. Therefore, 2 - 2 1/2 inch diameter pintles per bearing are adequate.

Note that this procedure is on the conservative side since the dead load friction is not considered in resisting part of the wind force. However, if dead load friction is considered, an assumption is required for wind uplift on the superstructure which tends to lower the dead load reaction.

| 27.6 BEARING SELECTION EXAMPLE, "A-T" BEARINGS

This bearing design example will illustrate the computation of loads and the selection of standard bearings. Figure the reactions on a 60 foot span consisting of steel beams at nine foot spaces carrying HS20 live loading on expansion bearings.

Est. Reactions Due to Dead Load 30(.90+.15+.05) = 33.0

Est. Reactions Due to FWS Load (30)(.18) = 5.4
38.4^k

HS20 live load reaction from the AASHTO Simple Span Table is 60.8^k due to truck loading. This loading is placed using two distribution factors. The first is for loading applied at the abutment. This is the simple beam distribution factor (1+5/9+3/9) = 1.89. The wheel load at the abutment equals 16^k. The in-span distribution factor is S/5.5 = 9/5.5 = 1.64. Impact is 30%.

Live Load Reaction is: = (16)(1.89)+(60.8/2-16)(1.3)
= (30.4+23.6)(1.3) = 70.2^k

Total Reaction, DL+LL = 38.4+70.2 = 108.6^k

If the bearing is to support a wide flange beam having a 12" flange, the selection is first
| based on flange width.

The expansion bearing selection for a 12" flange Standard 27.8 is as follows:

TYPE "A-T" EXPANSION, 12" WIDTH, HAVING A CAPACITY OF 140^K.

A computation of HS20 lane loading reaction is in order. For a 60 foot span, the uniform load is (30')(.64) = 19.2^k. The point load for shear is 26^k. The respective wheel loads are 9.6^k and 13^k. Live load reaction using our two distribution factors is (9.6)(.64+13)(1.89)(1.3) = 52.7^k.

It is apparent that lane loading produces a reaction lower than the 70.2^k due to truck loading.

| ## 27.7 DESIGN EXAMPLE - LAMINATED (STEEL REINFORCED) ELASTOMERIC BEARING

English units are used.

This design example is for a 3-span (3 @ 113 ft.) 70" prestressed girder structure. The piers are fixed supports and the abutments accommodate expansion.

Design Data
Total Expansion Length: 170 ft. (C_L bridge to C_L abut.)
Profile Grade Line: 0.5% (Constant).
| Bearing Location: (A-3) Abutments (East & West)
Girder Type: 70" Prestressed Girder
Bottom Flange Width (b_f): 2 ft.-2 in.
D.L. Reaction @ Brg.: 132^K (service load)
L.L. Reaction @ Brg.: 77^K (service load w/o impact)
| Low Temperature Zone: C (Southern Wisc.) - (see AASHTO Fig. 14.6.5.2-1)
| Durometer (55±5); Elastomer Grade (3) - (see AASHTO Fig. 14.6.5.2-1, Table 14.6.5.2-2)
| Shear Modulus (G): (112.5 p.s.i. $\leq$ G $\leq$ 165 p.s.i.) (Table 14.6.5.2-1)
Steel Reinf. Plates: ASTM A709 Grade 36

| Design Example is based on "Design Method A" (AASHTO 14.6.6)
For the research report leading to the development of Design Method A, see National Cooperative Highway Research Program Report #248 (NCHRP 248).

NCHRP 248 states that the primary mode of failure in the elastomer is from shearing stresses near the edge of the bearing that cause delamination from the steel reinforcement. Compression and rotation (which cause bulging of the elastomer) plus shear, all produce shear strain in the elastomer which cause diagonal tension strains. (See Figure below). The code requirements for allowable compressive stress, rotation and shear combine to keep tensile strain to a fraction of the elastomer's capacity.

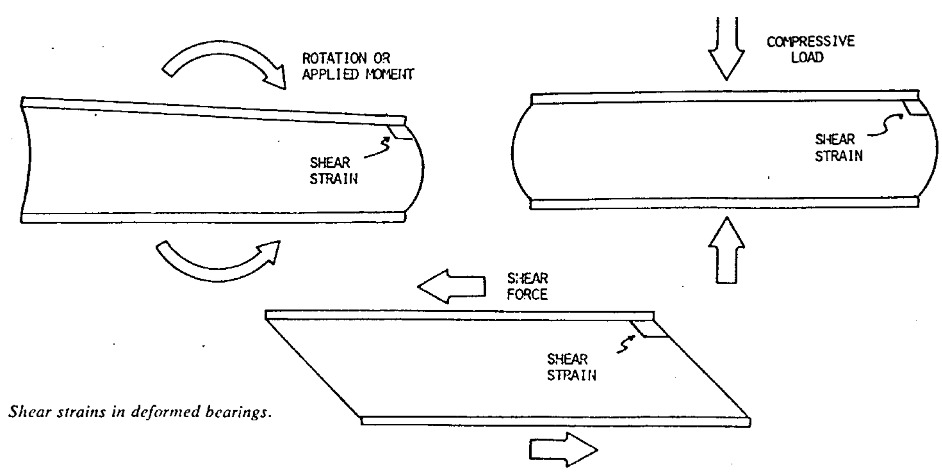


FIGURE 7.1

| • Shear (AASHTO 14.6.6.3.4)

Find horizontal bridge movement due to thermal and creep/shrinkage effects computed in accordance with Wisconsin Bridge Design Manual Chapter 28, Section 28.1(4).

- Temperature range for Prestressed Girders: +5°F. to + 85°F.
- Thermal coeff. of expansion for Prestressed Girders: 6 (10^{-6})ft/ft/°F.
- Creep/shrinkage coeff. for Prestressed Girders: $3(10^{-4})$ft/ft.

In AASHTO 14.6.6.3.4 and 14.4.1, the definition for shear deformation (Δs) of the bearing states that its contribution from thermal effects are computed between the installation temperature and the least favorable extreme temperature. In NCHRP 20-07/106, the installation temperature used for designing bearings supporting concrete superstructures is defined. As a result of this report, Bridge Standard 27.7 and this example are based on a design installation temperature of 60°F (see Section 27.2). Therefore the maximum design temperature range is:

$$\Delta T = 60°F. – (+5°F.) = 55°F.$$

$$\begin{aligned}\Delta s_T &= \text{(Expansion length) (coeff. of expansion)} (\Delta T) \\ &= (170\text{ ft.})(.0000060)(55°F.) \\ &= 0.056\text{ ft.} = 0.672" \text{ (due to thermal effects)}\end{aligned}$$

Any other conditions that may contribute to shear deformation of the bearing such as creep/shrinkage effects should be combined with the (Δs_T) due to thermal effects to produce a value for total shear deformation.

$$\begin{aligned}\Delta s_{cr/shr} &= \text{(Expansion length)(creep/shrinkage coeff.)} \\ &= (170\text{ ft.})(.0003\text{ ft./ft.}) \\ &= 0.051\text{ ft.} = 0.612\text{” (due to creep/shr. effects).}\end{aligned}$$

Then total shear deformation: $\Delta s = \Delta s_T + \Delta s_{cr/shr} = 0.672” + 0.612”$

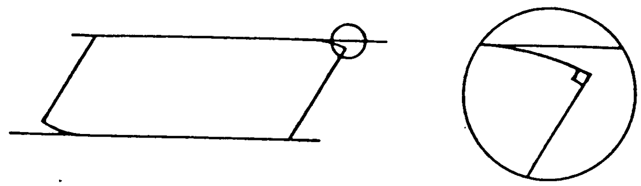
$$\therefore \Delta s = 1.28”$$

Therefore, from AASHTO 14.6.6.3.4

$$\underline{h_{rt}\text{ (min.)}} = 2\ (\Delta s) = (2)(1.28") = \underline{2.56"}$$

where h_{rt} = total thickness of all elastomer layers

NCHRP 248 states that the reason shear deformation (Δs) is limited to 0.5 (h_{rt}) is to avoid rollover at the edges and prevent delamination. (See Figure 7.2).



Rollover at edge of bearing.

FIGURE 7.2

Looking at Bridge Standard 27.7 and its Table used for preliminary design, we could have used our expansion length of 170 ft. and directly found a value for (h_{rt}) in column 4 of this Table equal to <u>3</u>".

$\therefore$ <u>h_{rt} = 3".</u>

• <u>Stability</u> (AASHTO 14.6.6.3.6)

To ensure stability of the bearing the following requirements must be met:

$$L \geq 3(T)$$
$$W \geq 3(T)$$

where L = gross bearing dimension in longitudinal direction
W = gross bearing dimension in transverse direction
T = total height of bearing (sum of thickness of all steel reinforcement layers and all elastomer layers).

From Table on Bridge Standard 27.7 we find that; h_{rt} = 3", steel reinforcing plate thickness = 1/8", thickness of internal elastomer layer = 1/2", thickness of cover elastomer layer = 1/4" and the total bearing height (T) = 3 3/4".

Therefore,

$$W(\text{min.}) = 3(T) = 3\ (3\ 3/4") = 11\ 1/4"$$

We see on Bridge Standard 27.7 that (W) is equal to the girder bottom flange width (b_f) minus (2").

$$W = b_f - 2" = (26")-(2") = 24" > 11\ 1/4".$$

$\therefore$ <u>W = 24"</u>

$$L(\text{min.}) = 3(T) = 3(3\ 3/4") = 11\ 1/4"$$

$\therefore$ $\underline{L = 12"}$

- Compressive Stress (AASHTO 14.6.6.3.2)

The average compressive stress due to total dead plus live load (σ_{TL}) in any layer shall satisfy:

σ_{TL} (total load) $\leq G_{min}(S)$

and also σ_{TL} (total load) $\leq$1,000 p.s.i. (for steel-reinforced bearings).

where $G_{min.}$ = 112.5 p.s.i., and (S) is the Shape Factor for the thickest elastomer layer in the bearing.

$$\text{Shape Factor } (S) = \frac{\text{Plan Area}}{\text{Area of Perimeter Free to Bulge}} = \frac{(L)(W)}{(2)(h_{r\ max})(L+W)}$$

where $h_{r\ max}$ = thickness of thickest elastomer layer.

The Shape Factor(S) accounts for the difference between plain and reinforced elastomeric bearings by adjusting the area that is effective in compression due to bulging of the perimeter of the elastomer.

We know L = 12", W = 24", $h_{r\ max}$ = 1/2" (internal layer)

$$S(\text{internal layer}) = \frac{(12")(24")}{(2)(1/2")(12" + 24")} = 8.0$$

(Internal layer): σ_{TL}(total load) $\leq G_{min}(S)$ = (112.5 p.s.i.)(8.0) = 900 p.s.i.,

this controls over σ_{TL} (total load) < 1,000 psi

$\cong$ Internal layer: σ_{TL} (total load) $\leq$ 900 p.s.i.

$$\sigma_{TL}\ (\text{total load}) = \frac{\text{D.L. Reaction + L.L. Reaction}}{(L)(W)}$$

$$\cong \sigma_{TL}\ (\text{total load}) = \frac{(132^{k.} + 77^{k.})(1{,}000)}{(12")(24")} = 726 \text{ p.s.i.} < 900 \text{ p.s.i. O.K.}$$

• Compressive Deflection (AASHTO 14.6.6.3.3)

The compressive deflection (δ), of the bearing shall be limited to ensure the serviceability of the bridge. Deflection should be limited to ensure that deck joints and seals are not damaged. Relative deflections across joints must be restricted so that a step doesn't occur at a deck joint. AASHTO 14.6.5.3.3 recommends that a maximum relative deflection across a joint be limited to (1/8").

Deflections due to service loads are calculated as:

$$\delta = \sum \varepsilon_i h_{ri}$$

where (h_{ri}) equals the thickness of elastomer layer (i), and (ε_i) is the compressive strain in elastomer layer (i) and can be found for our example using AASHTO Figure 14.6.5.3.3-1.

Calculate (ε_i) by examining dead load, live load, and creep either in appropriate combination or individually, whichever would create the largest relative deflection across a joint.

h_{ri}(internal layer) = 1/2" , h_{ri}(cover layer) = 1/4"

Therefore, Shape Factors(S) for use in Figure 14.6.5.3.3-1 are:

(internal layer)-S = 8.0
(cover layer)-S = 16.0

The average compressive stress due to service loads are:
σ_{TL}(total load) = 726 p.s.i.
σ_D(dead load) = 458 p.s.i.
σ_L(live load) = 268 p.s.i.

Check relative deflection across the joint at the abutment, for dead load, live load and creep.

Using dead load compressive stress, shape factor and Figure 14.6.5.3.3-1,

ε_i(internal layer) = .023
ε_i(cover layer) = .018

Therefore,

$$\delta \text{ (dead load)} = \sum_i \varepsilon_i h_{ri} = (2)(\varepsilon_i\text{-cover})(h_{ri}\text{-cover})+(5)(\varepsilon_i\text{-internal})(h_{ri}\text{-internal})$$

$$= (2)(.018)(.25")+(5)(.023)(.5")$$
$$= 0.066"$$

| Using live load compressive stress, shape factor and Figure 14.6.5.3.3-1,

| ε_i(internal layer) = .013
| ε_i(cover layer) = .011

Therefore,

| δ (live load) = $\sum_i \varepsilon_i h_{ri}$ = (2)(ε_i-cover)(h_{ri}-cover)+(5)(ε_i-internal)(h_{ri}-internal)

| = (2)(.011)(.25") + (5)(.013)(.5")

= 0.038"

| Considering creep, and using Table 14.6.5.2-1, we see creep deflection to be 35% of
| δ (dead load).

| δ (creep) = 35%(δ (dead load))
| = 0.35(0.066") = 0.023"

Therefore,

| δ (D.L. + L.L. + Creep) = 0.066" + 0.038" + 0.023"
| = 0.127" ≈ (0.125" = 1/8") O.K.

| Because the joint placement @ abut. backwall is completed after the superstructure dead load is in place, it would be more appropriate to check relative deflection across the joint for live load and creep.

Therefore,

| δ (L.L. + Creep) = 0.038" + 0.023"
= 0.061" < (0.125" = 1/8") O.K.

| • Reinforcement (AASHTO 14.6.6.3.7)

Reinforcing steel plates increase compressive and rotational stiffness, while maintaining flexibility in shear. The reinforcement must have adequate capacity to handle the tensile stresses produced in the plates as they counter the lateral bulging of the elastomer layers due to compression. These tensile stresses increase with compressive load.
| The thickness of the reinforcement, (h_s), shall satisfy the requirements of AASHTO
| 14.6.5.3.7.

| $h_s > \dfrac{3.0 h_{r\max} \sigma_{TL}}{F_y}$

where $h_{r\,max}$ = thickness of thickest elastomer layer

σ_{TL} = average compressive stress due to total dead plus live load

F_y = 36 ksi for ASTM A709 Grade 36 reinf.

$$\therefore h_s = \frac{3.0(0.5")(726\ psi)}{36{,}000\ psi} = 0.030" < 1/8"\ (O.K.)$$

and

$$h_s > \frac{2.0\ h_{r\max}\ \sigma_L}{F_{sr}}$$

where σ_L = average compressive stress due to live load
F_{sr} = allowable fatigue stress range for over 2 million cycles per AASHTO 10.3.1

$$\therefore h_s = \frac{2.0(0.5")(268\ psi)}{24{,}000\ psi} = 0.011" < 1/8"\ (O.K.)$$

• Rotation (AASHTO 14.6.6.3.5)

The limits on rotation (θ_m) in this section imply no net upwards displacement of any point on the bearing, in order to prevent tensile strains from occurring. The angle between the alignment of the underside of the girder (due to the slope of the gradeline, the rotation of the girder due to dead load plus live load, and camber) and the alignment of the bottom of the bearing (due to construction tolerances) is defined as rotation (θ_m) (NCHRP 248, Chapter 7 and Appendix D; 1985, 1992 AASHTO Commentaries), when the 1 1/2” top steel plate is not tapered. See Figure 7.3.

Elastomeric bearings must be placed on a level surface otherwise gravity loads will produce shear strain in the bearing due to inclined forces.

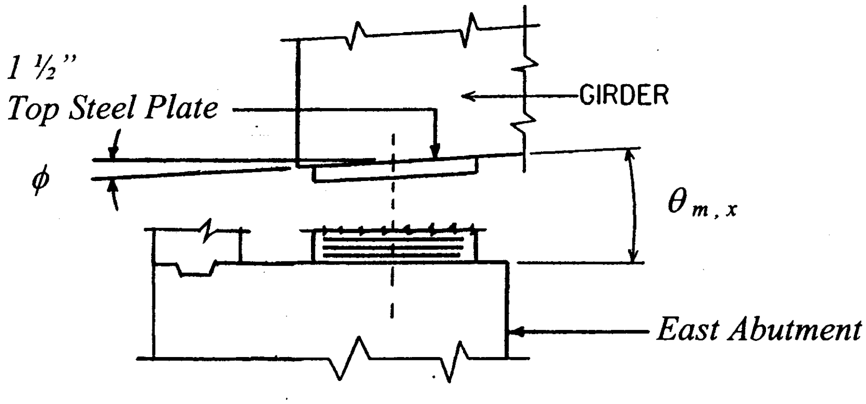


FIGURE 7.3

At <u>East Abutment</u>

Check requirement for a tapered top steel plate, AASHTO 14.7.2. The <u>angle</u> between the <u>alignment</u> of the <u>underside of the girder</u> (due to the slope of the gradeline, camber and dead load rotation) and a <u>horizontal line</u> is defined as (ø). (See Figure 7.3)

If (ø) exceeds (0.01 radians), the 1 1/2” top steel plate (as shown on Standard 27.7) is to be tapered to provide a level load surface along the bottom of this plate under these conditions. The tapered plate will have a minimum thickness of 1 1/2”. (See Figure 7.4).

$ø_{GL}$ (due to grade line) = +0.005 radians
$ø_{C/DL}$ (due to camber and dead load rotation) = +0.0025 radians
 $Ø_{actual} = Ø_{GL} + Ø_{C/DL} = 0.005 + 0.0025 = 0.0075$ radian < 0.010 radians
 No tapered plate req’d. for these conditions.

<u>Note</u>: If a tapered plate had been used, then the rotation (Θ_m) would be defined as, the <u>angle</u> between the <u>underside of tapered plate</u> (due to live load rotation) and the <u>bottom of the bearing</u> (due to construction tolerances). (See Figure 7.4).

Next check rotation limits for (Θ_m) at <u>East Abutment</u>.

$$\theta m,x = \frac{(2)\sigma_{TL}(n)}{G_{\max} S}(h_{ri}/L)^2$$ (about transverse axis)

$$\theta m,z = \frac{(2)\sigma_{TL}(n)}{G_{\max} S}(h_{ri}/W)^2$$ (about longitudinal axis)

n = number of interior layers of elastomer, where interior layers are defined as those layers which are bonded on each face. Because the top cover layer of elastomer is bonded on both faces, this will be included as 1/2 of a layer.

G_{max} = 165 psi.
S = shape factor of interior layer
σ_{TL} = average compressive stress due to total load (dead + live)
h_{ri} = thickness of ith elastomer layer

Therefore, limits are:

$$\Theta_{m,x}\ (\text{allow.}) = \frac{(2)(726\ psi)(5.5)}{(165\ psi)(8.0)}(0.5''/12'')^2 = 0.0105\ radians$$

$$\Theta_{m,z}\ (\text{allow.}) = \frac{(2)(726\ psi)(5.5)}{(165\ psi)(8.0)}(0.5''/24'')^2 = 0.0027\ radians$$

Rotations at Elastomeric Bearing are;

Θ_{GL} (due to grade line) = +0.005 radians
$\Theta_{C/DL}$ (due to camber and dead load rotation) = +0.0025 radians.
Θ_{LL} (due to live load rotation) = -0.002 radians
$\Theta_{m,x}$ (actual) = Θ_{GL} + $\Theta_{C/DL}$ + $\Theta_{LL.}$ = 0.005+ 0.0025–0.002 = 0.0055 radians
□ $\theta_{m,x}$ (actual) ≤ $\Theta_{m,x}$ (allow) (O.K.)
$\theta_{m,z}$ – (O.K.)

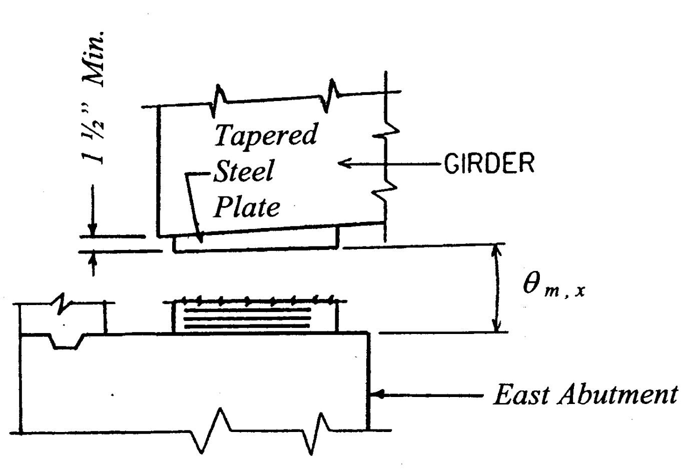


FIGURE 7.4

West Abutment

At West Abutment check requirement for tapered plates, AASHTO 14.7.2.

$Ø_{GL}$ (due to grade line) = +0.005 radians
$Ø_{C/DL}$ (due to camber and dead load rotation) = -0.0025 radians.
∴ $Ø_{actual} = Ø_{GL} + Ø_{C/DL} = 0.005 – 0.0025 = 0.0025$ radians < 0.010 radians
∴ No tapered plate req'd. for these conditions.

Next check rotation limits for (θ_m) at West Abutment.

$\theta_{m,x}$ (allow.) = 0.0105 radians
$\theta_{m,z}$ (allow.) = 0.0027 radians

Rotations at Elastomeric Bearing are:

θ_{GL} (due to grade line) = +0.005 radians
$\theta_{C/DL}$ (due to camber and dead load rotation) = -0.0025 radians
θ_{LL} (due to live load rotation) = +0.002 radians
$\Theta_{m,x}$ (actual)=$\Theta_{GL} + \Theta_{C/DL} + \Theta_{LL}$ = 0.005–0.0025+0.002 = 0.0045 radians
∴ $\theta_{m,x}$ (actual) ≤ $\Theta_{m,x}$ (allow.) (O.K.)
$\theta_{m,z}$ – (O.K.)

• Anchorage (AASHTO 14.7.3)

Elastomeric bearings may be left without anchorage if an adequate friction force is available to resist the horizontal force at the bottom of the bearing (AASHTO 14.7.3 Commentary).

The friction coefficient between elastomers and contact surfaces varies with compressive force and contact surface type. However in this section the coefficient of friction (μ = 0.2 or 1/5) is used as an approximation to check the need for anchorage of the bearing.

If the design shear force, (H_m), due to bearing deformation exceeds 1/5 (0.2) of the minimum vertical force, the bearing shall be secured against horizontal movement (AASHTO 14.6.6.4).

Check the design shear force (H_m) as defined in AASHTO 14.5.3.1, which is a function of elastomeric bearing deformation (Δ s) and compare with the friction force (P_D)/5 due to dead load alone.

$$H_m = \frac{G_{max}(L)(W)(\Delta s)}{(h_{rt})} = \frac{(165\text{ p.s.i.})(12")(24")(1.28")}{(3")(1000)} = 20.3^{k.}$$

and $\frac{P_D}{5} = \frac{132^{k.}}{5} = 26.4^{k.}$

Therefore, $(H_m) < P_D/5$, and anchorage is not required.

• Elastomeric Bearing summary:

ENGLISH	METRIC
L = 12"	300 mm
W = 24"	610 mm
Total Bearing Height = 3 3/4"	95 mm
(6) - 1/8" steel reinforcing plates	3 mm
(5) - 1/2" internal elastomer layers	13 mm
(2) - 1/4" elastomer cover layers	6 mm

• See AASHTO Section 14 (Division I) and Section 18 (Division II) for Elastomeric Bearing requirements.